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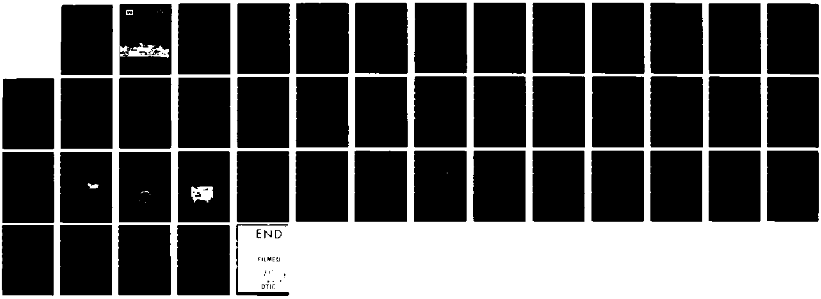

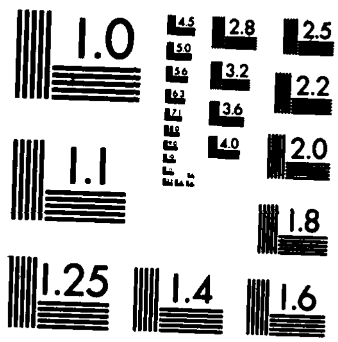

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IN-PLACE STABILITY AND DETERIORATION OF STRUCTURES

by

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Most old lock and dam structures do not meet current design requirements for stability. Since it is important that current stability requirements are met to assure structural safety, most of these structures must be modified to increase their resistance to sliding and overturning. These modifications are very expensive and could possibly be avoided if a more exact stability evaluation could be determined.

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20. ABSTRACT (Continued).

The stability of structures is presently evaluated by conventional rigid body stability computations, which require many assumptions that may be too conservative. For example, the base of the structure is considered flat when, in fact, it may be irregular and keyed into the foundation giving a much greater resistance to sliding. A procedure for determining in-place stability could be very attractive and cost-effective.

A good background in stability evaluation is a prerequisite to beginning in-place stability studies. A brief summary of conventional stability evaluations and some comments about their strengths and weaknesses are presented.

This study verified that in-place stability evaluations can be made. Tests were performed to evaluate in-place stability in the frequency domain. The only realistic evaluation was to determine displacement at zero frequency ($\omega = 0$, static condition). This was not successful because the equipment available for this study would not allow a good definition of D/F close to $\omega = 0$. Other equipment and some development of measurement techniques would allow a good definition of D/F close to $\omega = 0$. The time domain was then investigated and an in-place stability relationship was determined. The ratio of peak dynamic displacement and force, when plotted against ultimate static sliding resistance, gave a good relationship for a wide variety of interface conditions. Some details of this relationship should be evaluated but are beyond the scope of this study.

When considering changes in the condition of structures, the first concep in the evaluation is the effect of boundary conditions on evaluating parameters. All structures have interface (boundary) conditions, for example, a structure founded on rock. If the possible magnitude of these boundary conditions is not known, one will not know what parameter change is due to changes in the boundary condition or the structure itself. The base boundary condition for excitations parallel to the base is the main one associated with a lot of lock and dam monoliths and is the only boundary condition investigated in this study. In the past, the main parameter that has been used to obtain an indication of the changed condition of a given structure is its natural frequency. The natural frequency is not affected by significant changes in the base condition of the structure. The damping is affected by boundary conditions, but is also affected by many other parameters which will need to be investigated in more detail in the future.

PREFACE

This investigation was performed in the Structures Laboratory (SL), U. S. Army Engineer Waterways Experiment Station (WES). The study was authorized for initiation by WESVB DF dated 2 December 1980, subject, "Authority to Initiate New Starts on Projects A91D, AT22, and AT40," and was funded by the Assistant Secretary of the Army (R&D), under Project No. 4A161101A91D, Task 02, Work Unit 145.

The report was prepared by Dr. Carl E. Pace of the Research Group, Concrete Technology Division (CTD), and Mr. A. Michel Alexander of the Evaluation and Monitoring Group, CTD. Messrs. Dale Glass and Dan Wilson helped with the instrumentation and testing.

The study was conducted under the general supervision of Messrs. Bryant Mather, Chief, SL; W. J. Flathau, Assistant Chief, SL; and J. M. Scanlon, Chief, CTD, SL.

Commander and Director of the WES during the preparation and publication of this report was COL Tilford C. Creel, CE. Mr. F. R. Brown was Technical Director.

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CONVERSION FACTORS, NON-SI TO METRIC (SI)
UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
inches	25.4	millimetres
feet	304.8	millimetres
pounds (mass)	0.4535924	kilograms
pounds (force)	4.448222	newtons
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
pounds (force) per square inch	0.006894757	megapascals

IN-PLACE STABILITY AND DETERIORATION OF STRUCTURES

PART I: INTRODUCTION

Background

Conventional and in-place stability

1. New and old structures must have adequate stability. This means that these structures must meet present-day criteria for resistance to applied forces producing the effects of overturning, sliding, base pressures, and underseepage.

2. Because stability concepts seem simple, until recently they have not been given the consideration they warrant. This has resulted in deficiencies in the methods of evaluation. Inadequate consideration, incorrect assumptions about material properties, and deficiencies in stability evaluations have not caused many stability problems because of inherent factors of safety which are not considered in analysis. For example, in conventional stability computations, the monolith base is considered flat when, in fact, it may be irregular and keyed into the foundation, thereby producing a greater safety factor in sliding. However, inherent safety factors do not always exist; therefore, it is necessary to be conscientious and conservative in performing conventional stability evaluations.

3. Because of the inherent safety factors, many existing structures that do not meet present-day criteria, as evaluated by conventional stability methods, may actually be safe and may not require remedial strengthening. In-place stability evaluations would be helpful and cost-effective in delineating the structures which need remedial strengthening and those which are adequate as constructed, and also could serve as an independent check on conventional stability computations and assure that new structures are adequately stable.

4. Members of the staff of the Structures Laboratory (SL), U. S. Army Engineer Waterways Experiment Station (WES), have evaluated the

stability of existing lock and dam structures and recommended remedial measures. Their findings indicate that almost all of the existing lock and dam structures are inadequate in stability when analyzed by conventional methods and evaluated against current stability criteria. Remedial stability work is expensive; however, if the only means of evaluating stability indicates that the structure could be unsafe, then no alternative exists but to perform the required remedial work. This is true even if the lock or dam or both has experienced no indication of stability problems since it was constructed. The fact that the structure has had no problems is significant, but this is not sufficient to say that the structure will not fail due to some possible loading or deteriorating condition in the future. Conventional stability computations which indicate that a structure does not meet current requirements of stability criteria invariably leave the owner of the structure in a position of doubt, as far as the safety of the structure. For these reasons, it is important to be able to evaluate the in-place stability of structures.

Boundary effects on parameters indicating structure deterioration

5. Calculations of Young's modulus of elasticity (E) from dynamic tests have been made and used to indicate the changed state of integrity of concrete specimens in standard laboratory tests, such as those for resistance to freezing and thawing. The changes need not be caused by freezing and thawing. The dynamic E is a measure of the elastic qualities of the structure and is a good indicator of its structural integrity. For a particular structure and particular boundary conditions, the dynamic E depends on the natural frequency of the structure. Any deteriorating factor must affect the natural frequency of the structure in order to change the value of the dynamic E and, therefore, this change is an indication of deterioration.

6. Examining the parameters indicating deterioration for all types of structures, considering various geometries, assessing boundary conditions and material properties is an extensive program and beyond the effort of this study. This study limited its consideration to the effect of base boundary condition for excitation parallel to the base.

This is the most important boundary condition associated with most lock and dam monoliths.

7. When considering the changes or the condition of structures, the first concept in the evaluation is the effect of boundary conditions on evaluating parameters. All structures have interface (boundary) conditions, for example, a structure founded on rock. If the possible magnitude of these boundary conditions is not known, it will not be clear if parameter changes are due to boundary condition changes or due to changes in the structure itself. This study evaluates the sensitivity of the structure's natural frequency to changes in the base boundary condition of various structures.

Objective

8. The objective of this study was to verify that dynamic excitation methods using available transducers with measurement and analysis equipment can be used to determine the structural stability and integrity of existing structures.

Scope

9. This study used only available transducers and measurement and analysis equipment to verify that the in-place stability of structures can be determined by dynamic excitation. The sensitivity of a structure's natural frequency to the base-boundary condition from dynamic excitations parallel to the base was examined.

PART II: CONVENTIONAL STABILITY ANALYSIS

Introduction

10. To effectively verify in-place stability concepts, it is essential for the evaluator to have a good background in conventional stability computations, criteria development, and evaluation methods. There is no substitute for experience in practical stability evaluations and the responsibility of assuring the safety of field structures to highlight the strengths and weaknesses of evaluation methods. For these reasons, and since members of the staff of the SL have evaluated the stability of many field structures, a brief review of some of the basic stability concepts and evaluation methods is presented.

11. Structural stability is defined as adequate resistance against overturning, sliding, base pressures, and underseepage. Because stability concepts seem simple, in the past they have not been given the consideration which they warrant, and this has resulted in deficiencies in stability criteria. Some of the deficiencies in stability criteria have been addressed through practical stability evaluation in the SL, mainly through the CASE (Computer-Aided Structural Engineering) work, and through formalizing and extending the solutions to these deficiencies by the development of ETL 1110-2-256.

12. The criteria for evaluating the adequacy of the stability of structures are determined mainly by logical deduction and from past performance of structures which were designed using specific criteria. Even though stability criteria are not developed by rigorous mathematical and engineering analyses, the intuitive and logical deductions are just as important and should be developed without fallacies. Many assumptions are made in performing stability computations. In-place material properties must be approximated from sampling and laboratory tests. In many cases, the backfill is composed of sandy material filled with large rock and cobbles and cannot be adequately sampled and tested; therefore, the horizontal pressure it exerts on a lock wall has to be estimated. Interface properties have to be estimated. Attempts are made to make the

assumptions conservative, but in many cases trying to be conservative--but not too conservative--is not a clearcut decision. In special situations and because the material is sampled at intervals along the structure, the assumptions may be too conservative or insufficiently conservative. Inherent safety factors (such as assuming the monolith base is flat when, in fact, it may be irregular and keyed into the foundation) and conservatism in estimating material properties do not always have to be the case; therefore, it would be advantageous to have a way of evaluating the in-place stability of existing structures and checking the stability of newly built structures.

Sliding Stability Evaluations

13. Concepts, deficiencies, and suggested solutions which have been observed during practical stability evaluations by the members on the staff of the SL are outlined below.

Method 1

14. Ratio limit on $\Sigma H/\Sigma V$. The oldest criteria for evaluating the resistance to sliding are presented in EM 1110-2-2200 "Gravity Dam Design," and were intended primarily for concrete structures supported on competent rock foundations. Method 1 limits the ratio of horizontally applied forces to vertically applied forces $\Sigma H/\Sigma V$. Experience showed that structures on competent rock foundations are safe against sliding failures when the maximum ratio of $\Sigma H/\Sigma V$ is less than 0.65 for all static loading conditions and 0.85 for the normal operation plus earthquake loading conditions.

15. Deficiencies. The limit of the ratio of $\Sigma H/\Sigma V$ neglects cohesion. It uses resultant horizontally and vertically applied forces, and therefore is valid only for evaluating sliding surfaces along horizontal planes. This criterion gives a feeling for what designers of the past considered adequate; however, it is limited in its adequacy to give detailed evaluations for sliding stability and has not been used for many years.

Method 2

16. Shear-friction method (ETL 1110-2-184). The second method for evaluating sliding resistance is known as the shear-friction method and is presented in ETL 1110-2-22 and ETL 1110-2-184. It is defined as the horizontal resistance (R) to sliding divided by the horizontally applied loads (H).

$$S_{s-f} = \frac{R}{H} \tag{1}$$

When a downstream passive wedge contributes to the sliding resistance (P_p), the shear friction safety factor is defined as follows in ETL 1110-2-184.

$$S_{s-f} = \frac{R + P_p}{H} \tag{2}$$

17. The derivation of the general equation of horizontal sliding resistance to be used in shear friction Equations 1 and 2 is given below. The nomenclature for the shear-friction equations (ETL 1110-2-184) is:

- R = horizontal sliding resistance which can be mobilized along the critical path beneath the base of the wall. (It is applied as a driving force in the derivation to determine its value when it is just equal to the maximum horizontal components of the resisting forces which are created by only the vertical components (V) of the applied loads.)
- P_p = passive resistance of the earth or rock wedge adjacent to the wall
- ΣH = net applied horizontal driving force
- ΣV = summation of vertical applied forces above the assumed sliding plane which is below or at the base of the wall
- W = mass of downstream earth or rock wedge above inclined plane of resistance, plus any superimposed loads
- A = area of the potential failure path which develops the unit shearing strength. (Any portion of the assumed failure plane at the base-foundation interface which is not in compression should be excluded from A. However, if the assumed failure plane is not at the base-foundation interface but through the soil, no reduction in A should be made.)

ω = angle between the inclined failure path and a horizontal datum plane

C = cohesive strength = unit shearing strength at zero normal loading along the potential failure path beneath the base of the wall = test ultimate

ϕ = angle of internal friction of the foundation material (test ultimate value, degree) or, where applicable, the angle of sliding friction of the wall on the subgrade

FS = factor of safety

Notice that the horizontal components of the applied loads are not included in the derivation (Figure 1).

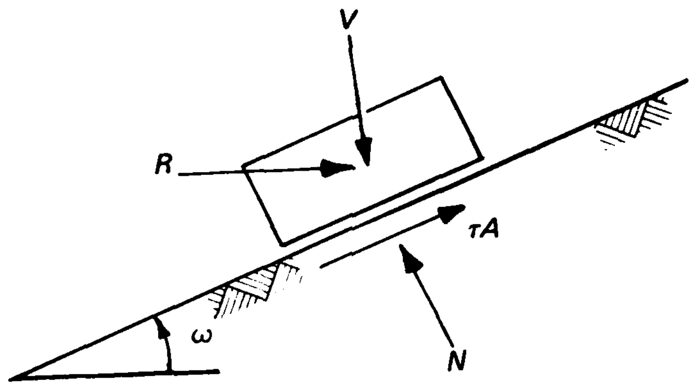


Figure 1. Vertical section through sliding mass on sloping surface

Sum vertical forces:

$$o = V - N \cos\omega + \tau A \sin\omega \tag{3}$$

where

$$\tau = C + \frac{N}{A} \tan\phi$$

then

$$o = V - N \cos\omega + CA \sin\omega + N \sin\omega \tan\phi$$

$$o = V - N (\cos\omega - \sin\omega \tan\phi) + CA \sin\omega$$

$$N = \frac{V + CA \sin\omega}{\cos\omega - \sin\omega \tan\phi} \tag{4}$$

Sum horizontal forces:

$$o = R - N \sin\omega - \tau A \cos\omega$$

$$o = R - N \sin\omega - CA \cos\omega - N \cos\omega \tan\phi$$

$$= R - N (\sin\omega + \cos\omega \tan\phi) - CA \cos\omega$$

Rearranging

$$R = N (\sin\omega + \cos\phi \cdot \tan\phi) + CA \cos\omega \tag{5}$$

Substitute Equation 3 into 4

$$R = \frac{V + CA \sin\omega}{\cos\omega - \sin\omega \cdot \tan\phi} (\sin\omega + \cos\omega \cdot \tan\phi + CA \cos\omega)$$

$$= \frac{V(\sin\omega + \cos\omega \cdot \tan\phi)}{\cos\omega - \sin\omega \cdot \tan\phi} + \frac{CA \sin\omega (\sin\omega + \cos\omega \cdot \tan\phi)}{\cos\omega - \sin\omega \cdot \tan\phi} + CA \cos\omega$$

Obtain a common denominator of the last two terms and reduce the equation to that given in ETL 1110-2-184.

$$R = \frac{V(\sin\omega + \cos\omega \cdot \tan\phi)}{\cos\omega - \sin\omega \cdot \tan\phi}$$

$$+ \frac{CA \sin^2\omega + CA \sin\omega \cos\omega \cdot \tan\phi + CA \cos^2\omega - CA \cos\omega \cdot \sin\omega \cdot \tan\phi}{\cos\omega - \sin\omega \cdot \tan\phi}$$

$$= \frac{V(\sin\omega + \cos\omega \cdot \tan\phi) \frac{1}{\cos\phi}}{(\cos\omega - \sin\omega \cdot \tan\phi) \frac{1}{\cos\phi}} + \frac{CA (\sin^2\omega + \cos^2\omega)}{\cos\omega (1 - \tan\omega \cdot \tan\phi)}$$

$$= V \tan(\phi + \omega) + \frac{1}{\cos\omega (1 - \tan\omega \cdot \tan\phi)} \tag{6}$$

18. This is the general equation for sliding resistance to be used in the shear-friction formula. The formulas for sliding uphill,

horizontally, and downhill and the passive wedge formula are specific cases of Equation 6. For uphill sliding ω is positive and Equation 6 is used directly. For horizontal sliding $\omega = 0$ and Equation 6 reduces to

$$R = \Sigma V\ (\tan\phi) + CA$$

For downhill sliding ω is negative, which reduces the equation to the following:

$$R = V\ [\tan\ (\phi - \omega)] + \frac{CA}{\cos\omega\ (1 + \tan\phi\ \tan\omega)}$$

The theoretical resistance offered by the passive wedge given in Equation 7 can be derived in several ways and is equivalent to Equation 6.

$$P_p = W\ \tan\ (\phi + \omega) + \frac{CA}{\cos\omega\ (1 - \tan\omega\ \tan\phi)} \tag{7}$$

19. These shear-friction formulas were used to evaluate sliding stability until members of the WES staff found deficiencies in the formulas and presented these deficiencies to the Office, Chief of Engineers (OCE) and other Corps offices. ETL 1110-2-184 was revised and has been superseded by ETL 1110-2-256.

20. The deficiencies in the sliding criteria as presented in ETL 1110-2-184 are many. If the deficiences in the shear-friction formula could be corrected and maximum sliding resistance divided by driving force used to evaluate sliding, there would still be some inconsistencies in the forces used for the stability evaluations. For example, the maximum passive resistance would be used in the sliding stability evaluation, but some effective passive resistance would have to be used in the overturning computations.

21. The deficiencies in the sliding friction method are presented below with some illustrative examples to clearly show the inadequacies in the evaluation method.

22. The main concern with the formulas in ETL 1110-2-184 is in relation to inclined planes.

23. <u>Deficiency 1.</u> Passive forces (any resistance forces not

included in the base resistance, such as strut resistance, etc.) which are not parallel to the sliding plane have a force normal to the sliding plane which affects the resistance to sliding. This effect is not considered by the ETL 1110-2-184 formulas. For example, if P_p (Figure 2) is at an angle to the sliding plane, it will have a component normal to the sliding plane which will reduce the summation of vertical forces and therefore reduce the frictional resistance forces which help keep the structure stable. This clearly illustrates that using maximum available passive forces which have a component that reduces frictional resistance is unrealistic.

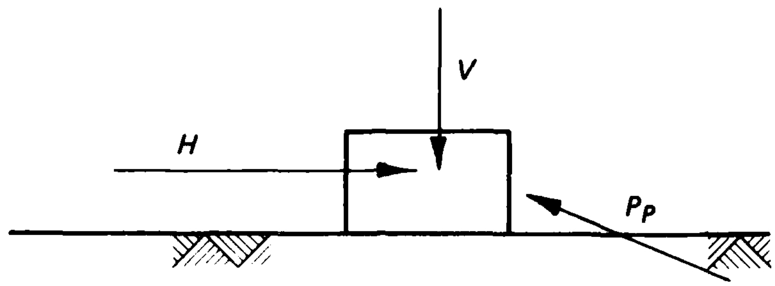
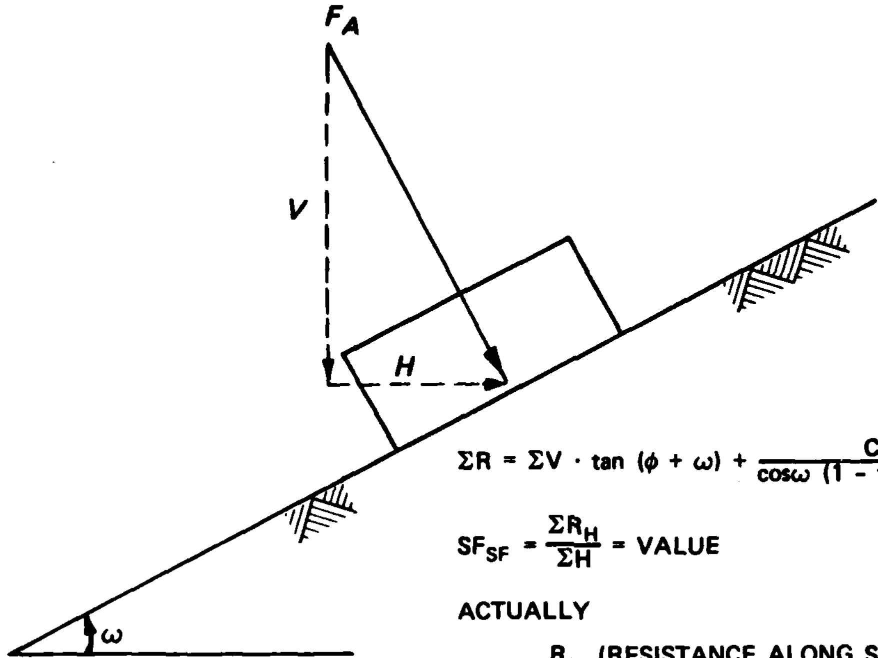


Figure 2. Passive force with a component normal to sliding plane

24. Deficiency 2. The safety factor for sliding is computed for an inclined sliding plane, but uses the ratio of horizontal force components. In reality, however, this safety factor should be calculated by using force components in the direction of the sliding plane.

25. The inadequacy in computing the safety factor against sliding, by dividing the horizontal components of resistance by the horizontal components of the driving forces, is best presented by an example of a structure on an inclined plane (Figure 3). Assume the resultant of all loadings (F_A) is normal to the sliding plane. Because we can have a V, H, ϕ, C, and ω, R_H can be a finite value in $SF_{SF} = \Sigma R_H / \Sigma H$. Using H, we can calculate a finite safety factor which can vary over a wide range depending on the values of V, H, ϕ, C, and ω. In reality if the resultant of the applied loads is normal to the plane of sliding, there is no tendency for the block to move and the safety factor is ∞.



$$\Sigma R = \Sigma V \cdot \tan(\phi + \omega) + \frac{CA}{\cos\omega\,(1 - \tan\omega \cdot \tan\phi)}$$

$$SF_{SF} = \frac{\Sigma R_H}{\Sigma H} = \text{VALUE}$$

ACTUALLY

$$SF_{SF} = \frac{R_\omega \text{ (RESISTANCE ALONG SLIDING PLANE)}}{0} = \infty$$

Figure 3. Illustration of sliding criteria limitation

26. The force vectors which drive and resist the movement of the structure are along the trial failure plane; therefore, the safety factor should be the ratio of the resisting forces to the driving forces in the direction of the inclined plane.

27. Deficiency 3. As the inclination of the failure plane (ω) approaches $90° - \phi$, the factor of safety approaches ∞. The safety factor of ∞ at $\omega = 90° - \phi$ is independent of the resistance parameters or applied forces when, in reality, the safety factor is dependent on these parameters and forces.

28. Deficiency 4, phase development of resistances. The various types of resistance which cause a structure to be stable (friction, cohesion, and passive) do not develop at the same rate in relation to the resultant applied load. Their comparative rates of development may be as shown in Figure 4.

29. The maximum magnitude of each of the various types of

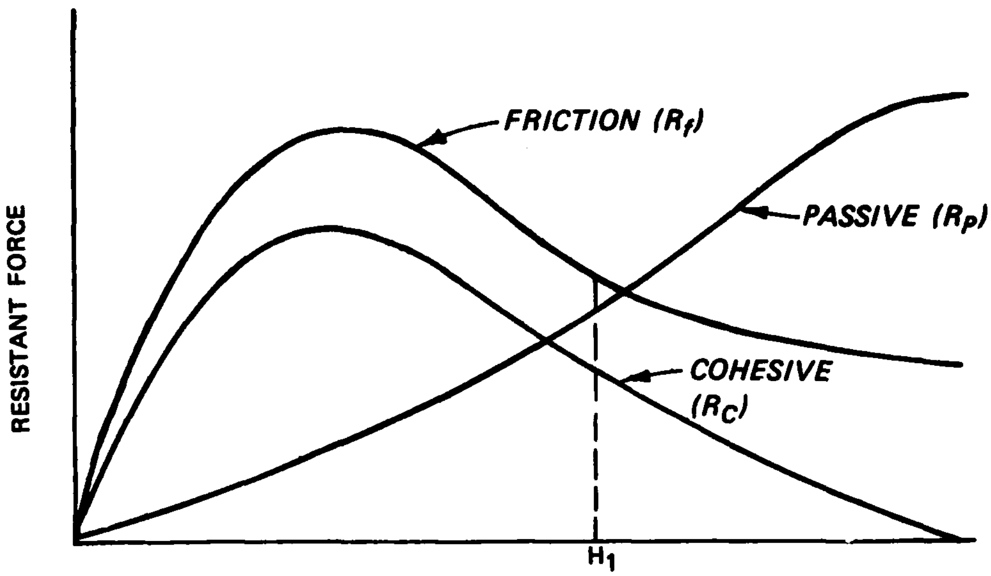


Figure 4. An assumed phase development of resistance

resistances can be computed in the conventional manner, but their developments, in relation to each other, may vary in phase. This is important because if the maximum of each resistance does not develop at the same resultant applied load, it will never be possible to have a total resistance equal to the sum of their maxima:

$$SF_{s-f} = \frac{R_H}{H} = \frac{R_f + R_C + R_p}{H}$$

At H_1 on the resultant applied load axis in the preceding formula, the total resistance would not be the sum of the maximums but would be the sum of the specific resistances at H_1. This could cause a significant effect on the sliding safety factor as the applied loads increase.

30. Deficiency 5. The maximum values of resistance along with the applied case loadings do not form a loading situation which is in equilibrium. Moment equilibrium is not satisfied.

Method 3

31. Force equilibrium method. The third method used to evaluate sliding criteria has been termed, "the allowable strength equilibrium method." The allowable strength equilibrium method is based on the

concept that the applied forces acting on the structure and the resistance forces are in force equilibrium. This is in accordance with reality and is a good way to approach sliding stability. This procedure for evaluating stability can be formulated and solved in several ways, but the basic idea will be presented here, and from this basic concept any of the other approaches can be understood. Only single plane sliding of the structure, on a horizontal base, is considered; if multiple plane sliding is needed, an extension of single plane sliding can be found in ETL 1110-2-256. If the sliding plane is at an angle, the force summations must be in the direction of the sliding plane. The same is true for horizontal planes but results in more complicated equations for inclined planes.

32. The first step is to set the driving forces to equal the resistance forces.

$$\Sigma F_D = \Sigma F_R$$

33. If the driving forces consist of some active soil or rock forces, they can be formulated by the appropriate geotechnical principles. The maximum resistance force for sliding along a horizontal plane is given by $V \tan\phi + CA$. The driving forces will then equal the resistance forces with only partial development of the maximum ϕ and C values (represented by ϕ' and C').

$$F_D = V \tan\phi' + C'A \tag{6}$$

The safety factors are then represented by

$$SF_1 = \frac{\tan\phi}{\tan\phi'} \tag{7}$$

$$SF_2 = \frac{C}{C'} \tag{8}$$

Assume $SF_1 = SF_2$ (other assumptions could be made). The solution of Equations 7 and 8 can be obtained by assuming a safety factor (SF) since the maximum values of friction and cohesion (ϕ and C) are known. Having obtained the developed strength values of ϕ' and C', Equation 6 can be solved. If $F_D \neq F_R$, another safety factor can be

selected and another ϕ' and C' solved until the driving and resisting forces are equal. The safety factor which makes the driving and resistance forces equal is the safety factor for this particular structural condition and case loading.

34. The rationale behind this method is presented as follows:

a. Assume a block on an inclined plane (Figure 5).

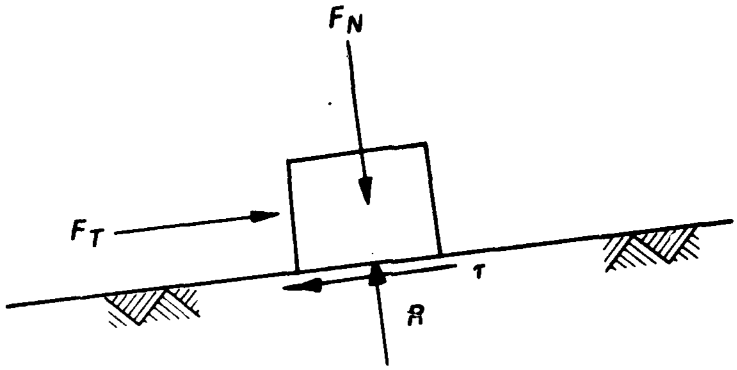


Figure 5. Block on an inclined plane

b. Assume tests have been made and the data are as presented below (Figure 6):

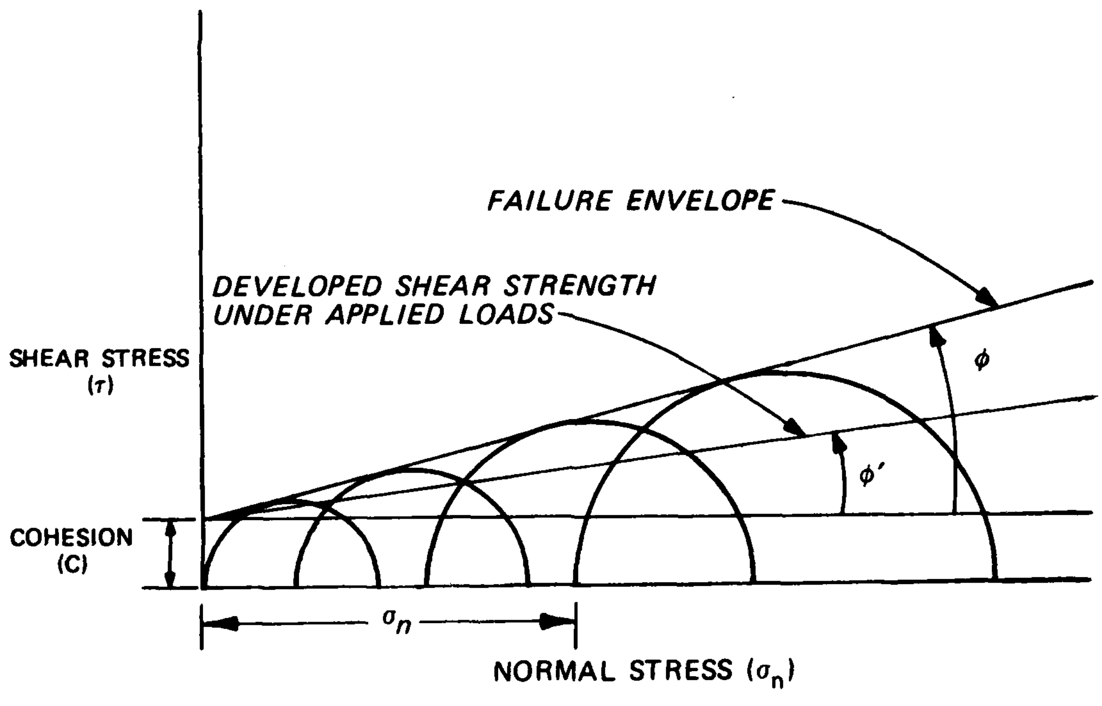


Figure 6. Triaxial test data

Neglecting cohesion:

$$\tan\phi = \frac{\tau_{max}}{\sigma_n} \quad \text{or} \quad \tau_{max} = \sigma_n \tan\phi$$

$$\tan\phi' = \frac{\tau_{developed}}{\sigma_n} \quad \text{or} \quad \tau_{developed} = \sigma_n \tan\phi'$$

$$SF = \frac{\tau_{max}}{\tau_{developed}} = \frac{\sigma_n \tan\phi}{\sigma_n \tan\phi'} = \frac{\tan\phi}{\tan\phi'}$$

c. The stress-strain diagram for concrete structures of various weights sliding on a rock foundation is presented in Figure 7.

35. As the weight of the structure (V) is increased, there will be more stress developed for a given value of strain. The safety factor definition, as presented in b, assumes that proportional positions (say one-half the maximum shear stress values produced by the various V's) on the stress-strain curve are related, just as the maximum values, by the tangent function. That is, they fall on a straight line when

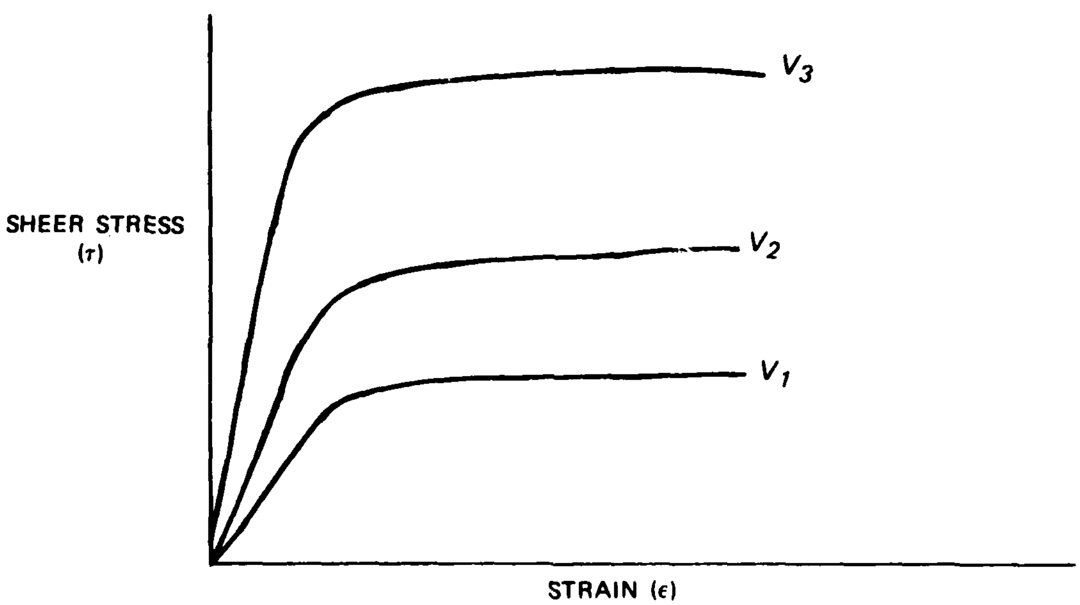


Figure 7. Typical stress-strain curve for concrete sliding on a rock foundation

plotted as shown by the developed shear strength line in Figure 6.

36. This definition makes no attempt to establish how the actual stress-strain curve for sliding failure of the block on the plane develops. As the developed shear strength line in Figure 6, it is related proportionally to values of the maximum shear stresses.

37. This is as good an assumption as any and is, at this time, the best method available for evaluating sliding stability.

38. It has been found from shear tests of concrete on various materials that the stress-strain curve for concrete on various surfaces does develop linearly to a maximum and then strain at constant load.

39. Deficiency. The main deficiency with the force equilibrium method is as presented in Deficiency 4, under the shear-friction method. The various types of resistance which cause a structure to be stable (friction, cohesion, and passive) do not develop at the same rate in relation to the resultant applied load. The sum of the individual maximums will never be possible, since the maximums of each resistance do not develop at the same resultant applied load. As factual information is developed about how the resistances develop with applied load, the sliding criteria can be modified to incorporate and account for the resistance developments. Another approach would be to incorporate interface properties into a finite-element analysis.

40. Compatibility of strains can be accounted for through finite-element modeling of the stability problem and using appropriate stress-strain characteristics of the materials.

Overturning Stability

41. For many years, the criteria for determining the adequacy of a structure in overturning stability have been evaluated by where the resultant of applied loads intersects the base of the structure. If the resultant falls within the middle third of the base, the total base will be in compression and the structure is safe against overturning. For certain loading conditions, the resultant can fall outside the middle third and the structure can still be judged as adequate. For

example, when "at-rest" earth pressures are used in normal operation, extreme maintenance, or maximum flood loading cases, the resultant of applied loads can fall outside the kern, but at least 75 percent of the base must be in compression. For operating conditions with earthquake, the resultant has only to fall within the base, but the allowable foundation stresses should not be exceeded.

42. The main problem with this evaluation method is that it does not consider the centroid of the structure and dead load. The deficiency is best illustrated by an example.

43. Consider simple overturning criteria (Figure 8) for a block acted on only by its own weight; the resultant acts through its centroid and at the center of the base width (B). It is very stable. If the base width is increased by adding, for example, a thin reinforced

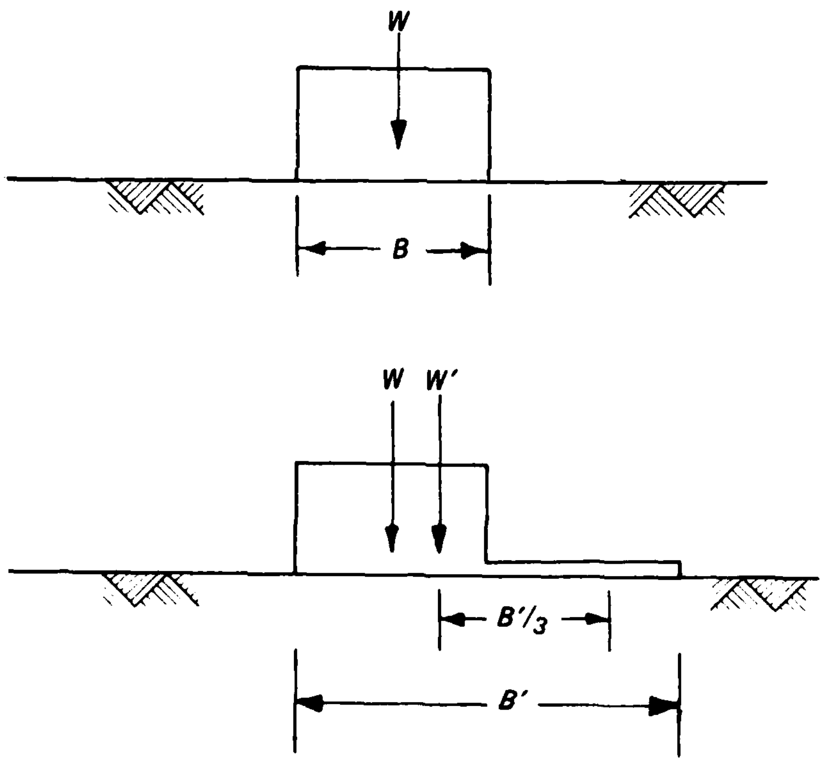


Figure 8. Example of limitation in overturning

section to the right, the base width is now B' . If the weight of the added base is small, relative to the original structure, such that the resultant of the total structure (W') is outside the middle third of B' to the left, by extending the concrete base, the stable structure is harder to turn over to the left; however, the middle third criteria depict the structure now unstable to the left. This is not logical.

44. It is suggested that the limits in which the resultant of applied loads intersect the base of the structure be used as evaluating criteria (except the limits be applied to the length of the base to either side of the centroid of the structure and dead load).

45. For example, if the centroid of the structure and dead load is at $B/4$, the resultant of applied load must fall in $(1/6)2B/4 = B/12$ to the left and $(1/6)(2)(3/4B) = B/4$ to the right of the centroid of the structure and dead load to represent the common middle third criteria.

Base Pressures

46. The actual base pressures (about one axis) are determined by the standard formula $p = (P/A) \pm (Mc/I)$ which assumes that the base and foundation are rigid where

- p = pressure at a particular point on base considering one axis through centroid of base
- P = normal load on base
- A = area of base
- M = moment about the axis through centroid of base
- c = distance from neutral axis perpendicular to axis being considered to points on base where base pressure is wanted
- I = moment of inertia of section

This assumption is usually adequate, but more precise results can be obtained if deformations of the structure and foundation are taken into account. The deformations can be taken into account by using finite-element analysis of the structural problem.

PART III: IN-PLACE STABILITY

Introduction

47. Only in-place stability concepts are being considered for evaluating sliding stability. The overturning resistance of structures can be adequately evaluated by conventional computations, if the evaluating criteria are modified to be realistic. Base pressure evaluations based on conventional computations are considered adequate and can be refined only by considering structure and foundation deformation properties by some procedure such as the finite-element analysis.

48. In all probability, many old structures which do not meet the present sliding stability criteria are actually adequate in resistance to sliding. At present, many of the older structures are being modified at substantial cost, because there are no means other than conventional computations to assure their safety in sliding. If a structure could be excited and its actual in-place resistance to sliding determined by using standard transducer and measuring and analysis systems, it would, in all probability, save much of the cost of remedial work.

49. The concept of determining the in-place sliding resistance of a structure was envisioned as being accomplished by determining the structure's displacement force ratio as a function of frequency, and then extrapolate this ratio to zero frequency. The ratio of displacement and force, at zero frequency, would then be used to determine the in-place stability of the structure. The ratio does not give directly the safety of the in-place structure in sliding because some criteria must be available to evaluate what this horizontal force-deflection relationship means in relation to sliding safety.

50. The way to evaluate this is not by developing new criteria which must be proven with time, but to relate the horizontal force-deflection relationship as determined in the field to conventional stability analysis in such a way as to determine the safety factor against sliding in relation to conventional sliding safety factor calculations. This relationship can be obtained by using the same laboratory

test data as those used in performing the conventional stability analysis computations in conjunction with in-place measurements. Laboratory tests are used to determine the angle of internal friction and cohesion of the weakest plane, or combination of planes below a structure. Shear tests which give these data also give the load-deflection characteristics of these shear planes.

51. The safety factor, as determined by the laboratory test data, can be ratioed by $\frac{\text{laboratory deflection/load}}{\text{field deflection/load}}$ to obtain the in-place factor of safety against sliding. This is saying that if the structure is more difficult to displace in the field than the laboratory test data indicate, it is safer in its resistance to sliding.

52. In the final analysis, the dynamic, maximum deflection/maximum force ratio versus static sliding failure load was found to have a consistent relation and can be used to determine in-place stability.

Experimental Setup for Verification of In-Place Stability

53. The properties of the foundation-structure interface determine the resistance which can be developed against structural sliding. For example, if a structure is on rollers it can be moved with a small force parallel to its foundation; whereas, if it is on a rough surface, it has much more sliding resistance. It is easily seen that in-place stability measurements must relate the dynamic excitation and response measurements to the resistance of the structure to slide (move) on its foundation. To verify in-place stability evaluations, it is important to know the actual static sliding failure load to correlate with the dynamic excitation and response measurements.

54. A study and evaluation of in-place stability can be made after sliding failure loads and dynamic excitations and response measurements for the same structures and base conditions have been determined. In this study, tests on a structure were made to determine the sliding failure load on a given surface; then, dynamic excitation and response measurements were made for the same structure and base condition. A wide variety of structures and base conditions were tested in

order to establish a general trend of structure response. The data were analyzed and a verification was made so that in-place stability evaluations are possible.

Sliding Failure Loads

55. Sliding failure loads were determined from results of tests using a load cell attached to a hydraulic ram and supported by a steel beam (Figure 9). The ram, when pushed by hydraulic pressure, pulls on

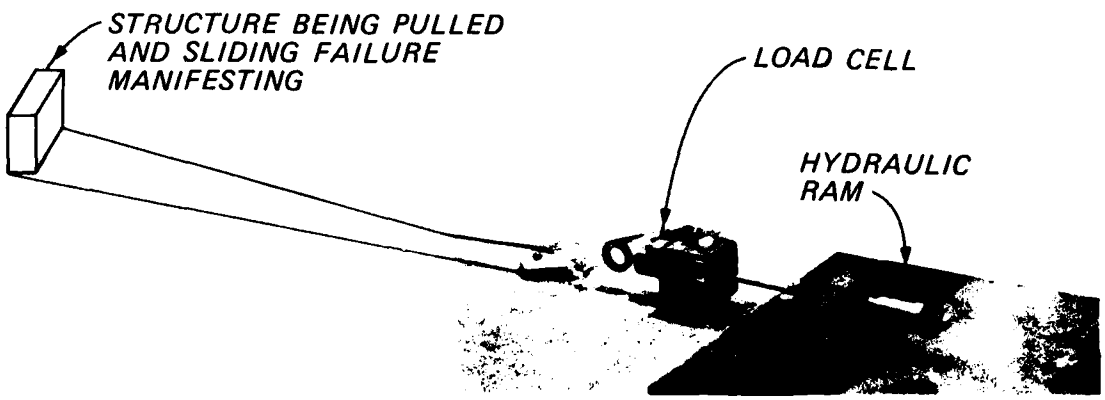


Figure 9. Overall view of experimental setup

the load cell and the structure, causing the structure to have deflections in sliding. The deflection for interface movement is linear (Figure 10) from zero to failure for almost all base conditions. The load cell measures the pull on the structure, and this pulling load is fed into and plotted through electronic equipment. Two LVDT gages (Figure 11) are placed, one on each side and behind the structure, to measure the structure deflections. The output of the LVDT gages is fed through electronic equipment to the plotter. The electronic equipment allows calibrations to be applied to load cell and LVDT responses, and along with appropriate scale factors, a plot is made of load versus deflection for each LVDT gage until the structure fails in sliding

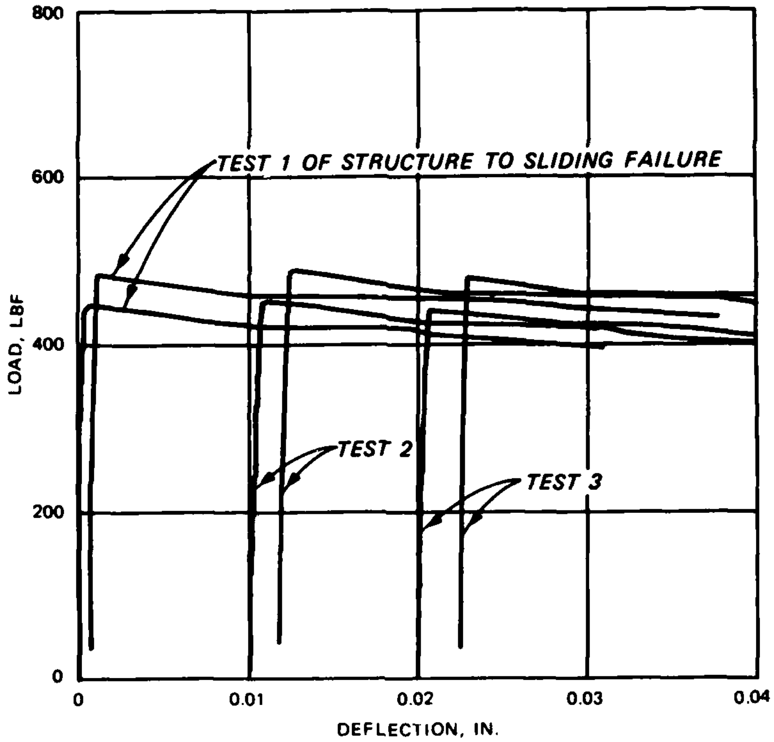


Figure 10. Typical deflections versus load of structure

Figure 11. LVDT gage for measuring structure deflections

(Figure 10). Several sliding failure loads are obtained and averaged, giving the average static sliding failure load of the structure.

Dynamic Excitation

56. The equipment which is available and most convenient to use in performing the measurement and analysis is a fast Fourier transform (FFT) analysis machine. The excitation can be performed by impacting the structure with a weight having a load cell behind its impact head. The load cell will measure the applied impact force, and an accelerometer at the structure's base will measure the structure's response. Accelerations or acceleration-force ratios can be integrated by the FFT machine to give displacements or displacement-force ratios. From previous experience, it is known that the accelerometer will make measurements of acceleration from which the displacement can be determined, and this can be done for very small values of displacement. The fast Fourier analysis machine used is a two-channel Hewlett Packard 5423 (Figure 12).

Figure 12. Fast Fourier analysis machine

57. The verifications of in-place stability measurements were started using the impedance values in the frequency domain of D/F (displacement/force); because, for a linear system these values do not vary with applied force. This will allow a unique transfer function to

be determined which will be valid for the structure. A/F (acceleration/force) is first obtained by the FFT machine and these data are integrated to obtain D/F .

58. Many tests and analyses were performed to try to determine other resistance parameters which could be used for the in-place stability evaluations. The following conclusions were reached:

a. The in-place stability criteria are not practical unless they can be related to the conventional stability safety factors, allowing for a determination of whether or not the conventional sliding safety factor increases or decreases.

b. Any dynamic resistance parameter, other than D/F (displacement/force), would require a development of criteria relating the parameter to static safety of the structure, and would be too extensive an effort to be practical. This necessitates using the D/F parameter.

c. A relation of the dynamic parameter D/F to the resistance to sliding of structures would be an excellent relation to the conventional sliding safety factor.

59. Obtaining a displacement-force ratio at zero frequency ($\omega = 0$, static condition) in the frequency domain was not successful because the equipment available for this study would not allow a good definition of D/F close to $\omega = 0$. Other equipment and some development of measurement techniques would allow a good definition of D/F close to $\omega = 0$.

60. The time domain was then investigated. A decision was made to investigate the ratio of maximum force (Figure 13) and maximum displacement (Figure 14) versus static sliding failure load of the structure. This relation seemed promising. This will be an ideal relation, if it exists, because the ratio of maximum deflection and maximum force obtained by a dynamic impact can be obtained for any field structure; and if from these values the probable failure load can be determined, the safety factor follows directly for each case loading.

61. Certain effects will have to be considered, such as the effect of impact height from the base of the structure, energy spectrum applied, and others. If in-place stability determinations can be verified, many of these factors must be evaluated in more detail,

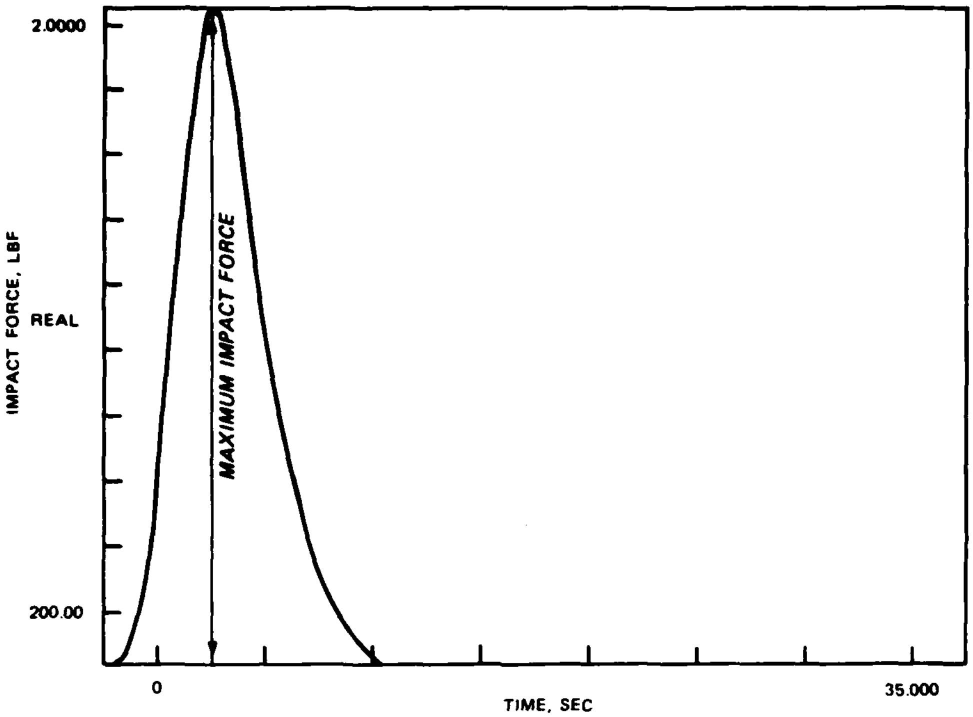


Figure 13. Typical impact force versus time record

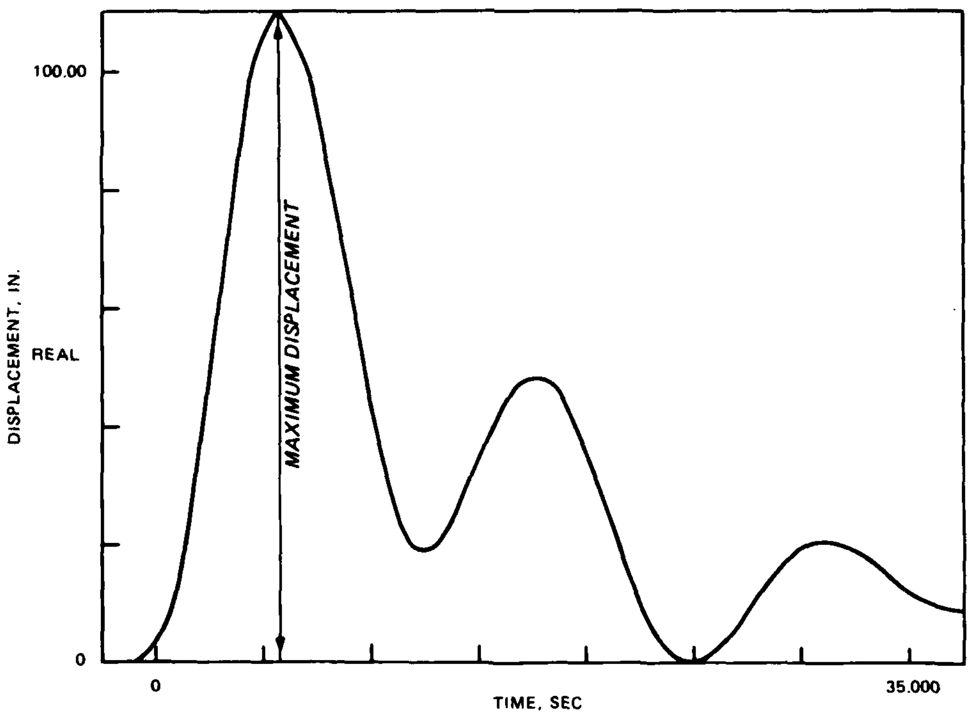


Figure 14. Typical displacement versus time record

because their detail evaluation is beyond the scope of this study.

62. The structural damping will not be appreciable for impact loads; therefore the maximum deflection of the structure will be a direct response from the impact excitation (Clough and Penzien*). The deflection of the structure at the base should be mainly a function of base resistance, excitation, and the distance above the base at which the structure is impacted. The relationship and verification of in-place stability measurements will be made only for impacts and response measurements at the structure base.

63. A good correlation was found between the ratio D_{max}/F_{max} (maximum deflection versus maximum impact force) and the static sliding failure load for various structures and base conditions. This relation is presented in Figure 15. Table 1 presents some facts about the structure and base conditions investigated. The equation for this relation is $D_{max}/F_{max} = 2.29 \times 10^{-4}(\text{sliding failure load})^{-1.12}$.

64. There are various parameters which will affect this relationship such as height of impact above structure base and impulse of the impact. These considerations should be made in more detail to refine the in-place stability evaluations.

65. The relation of D_{max}/F_{max} ratio to failure load is quite good when a soft impact hammer is used and the structure is hit at the base. The relation in Figure 15 definitely verifies that a structure can be excited in-place by a dynamic impact and the in-place stability of the structure determined.

66. The objective of this study was to verify in-place stability evaluations and this has been accomplished, as indicated by Figure 15. In the future, a refinement of this relation should be made by detailed determinations of variables (previously mentioned) and larger or more stable structures tested to extend the results.

67. After some experience with the in-place stability evaluations and their comparisons with conventional stability results, the in-place

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* Ray W. Clough and Joseph Penzien. 1975. Dynamics of Structures, McGraw-Hill.

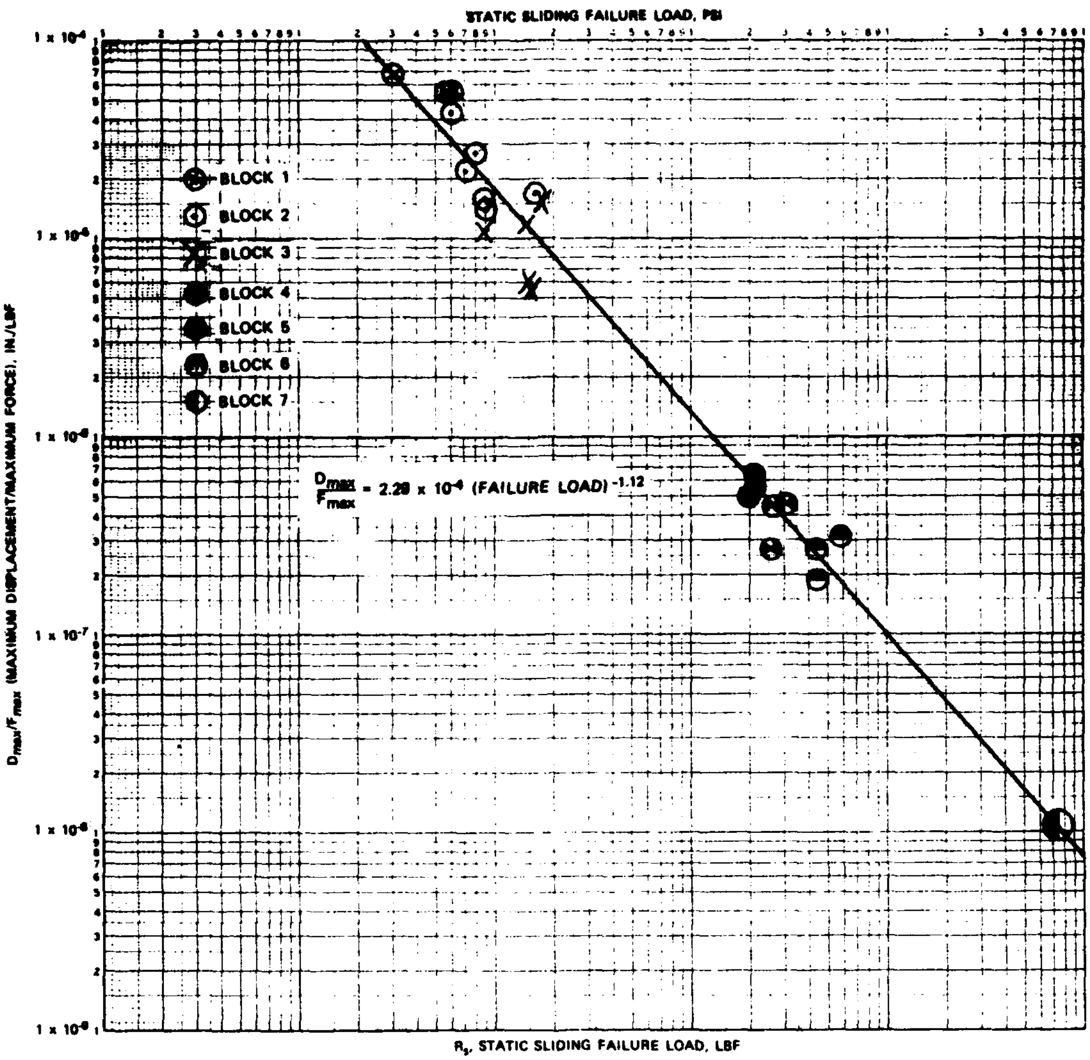

Figure 15. D/F versus static sliding failure load

stability evaluations can be made with confidence. Money can be saved by not having to do unnecessary remedial stability work.

Table 1

Structure and Base Conditions Investigated

Block No.	Size, in., and Mass, lb, of Concrete Block	Dimensions of Base of Block, in.	Sliding Foundation	Static Failure Load, R_s, lbf	Average $\frac{D_{max}}{F_{max}}$ in./lbf
1	3.5 × 4.6 × 7.27	3.5 × 7.27	Plexiglas	3.1	6.4×10^{-5}
	9.6		Sandpaper	6.0	5.3×10^{-5}
			Concrete	5.6	5.2×10^{-5}
2	3.5 × 4.55 × 13.6	3.5 × 13.6	Plastic coated plywood	7.2	2.2×10^{-5}
	18.5		Concrete	8.0	2.7×10^{-5}
			Adhesive Dow Corning 11 compound on Plexiglas	8.9	1.49×10^{-5}
			Fly ash on Plexiglas	6.0	4.2×10^{-5}
			Sandpaper	16.3	1.7×10^{-5}
			Aluminum sheet	8.3	1.53×10^{-5}
3	3.7 × 7.75 × 17.3	7.75 × 17.3	Aluminum sheet	14.9	6.0×10^{-6}
	40.2		Concrete	15.3	5.7×10^{-6}
			Plastic coated plywood	8.9	10.9×10^{-6}
			Fly ash on plastic coated plywood	14.6	11.8×10^{-6}
			Adhesive Dow Corning 11 compound on Plexiglas	18	16.0×10^{-6}
4	12.1 × 15.4 × 24.1	15.4 × 24.1	Sheet metal	200	5.0×10^{-7}
	396		Plastic coated plywood	212	6.5×10^{-7}
			Concrete floor	213	5.5×10^{-7}
5	17.87 × 17.87 × 26.25	17.87 × 26.25	Concrete	260	2.7×10^{-7}
	774		Plastic coated plywood	313	4.7×10^{-7}
			Steel plate	265	4.7×10^{-7}
6	20.25 × 24 × 41.25	20.25 × 24	Sheet metal	444	1.94×10^{-7}
	1742		Concrete floor	445	2.7×10^{-7}
			Plastic coated plywood	588	3.1×10^{-7}
7	36 × 72 × 120	36 × 72	Concrete floor	7500	1.05×10^{-8}
	28,000				

PART IV: DETERIORATION/INTEGRITY EVALUATION

Introduction

68. Because boundary conditions are so critical in the in-place stability of a structure, and because they directly affect the vibrational signature of a structure, it was felt that a nondestructive dynamic test of a structure could be used to determine how sensitive the structure's response is to boundary conditions.

69. The investigation of a structure's vibrational characteristics such as frequency, damping, and mode shape is called modal analysis. Modal analysis is the process by which the dynamics of an elastic structure is characterized. We would like to find the relationship between a structure's boundary conditions and changes in the structure's dynamic (modal) properties.

70. There are a number of factors that influence the modal properties. The relationship is complex. One can experimentally observe, however, broad changes in the modal characteristics by looking at three main factors: geometry, modulus, and boundary conditions.

71. Members of the WES staff routinely make resonant frequency measurements on concrete beams subjected to test for resistance to freezing and thawing. By controlling the geometry and the boundary condition of the specimen, it is known that the resonant frequency of a particular mode shape will decrease, as the modulus of elasticity of the specimen decreases.

72. It is known that a geometry change will affect the modal properties. If all things are maintained equal, an increase in length will decrease the resonant frequency of the fundamental flexural mode. If the thickness is increased, this will increase the frequency of vibration.

73. If a specimen is supported by narrow or knife-edge contacts at the nodes of vibration (for that particular mode), it will experience a minimum of damping. The largest amplitudes of vibrations are produced when supported in this fashion. Any other type of support that

restricts the movement at the antinodes increases the damping. A beam, for example, excited in a free-free mode (unrestrained at both ends) will develop a different mode shape than one that is in a fixed-free condition.

74. It is evident that many factors can influence changes in modal properties, and it must be determined at the time of measurement whether or not a change has occurred in the geometry and modulus. If boundary condition effects are to be determined these two factors should have not changed since the last measurement.

75. Mathematical modeling is not accurate enough to evaluate the base condition from a measurement of the modal properties; therefore, developing historical data would be the best approach. A measurement would be made at the time of construction (when it is known that the foundation is satisfactory), and then compared with later measurements to monitor changes. Modulus and geometry would have to remain constant or, if not, the influence of any change in either would have to be related to a change in the modal properties, making possible an evaluation of boundary conditions.

76. An excellent survey, by Structural Measurements Systems, Inc. of Santa Clara, California, on detecting damage of structures by measuring changes in their modal properties is referenced by Richardson.* Richardson said: "The underlying assumption of this survey is that changes in modal parameters are a reliable (and sensitive) indicator of changes in structural integrity. It is our contention, of course, based upon approximately 40 years of combined modal testing experiences, that this is indeed the case." Numerous references are given in this survey that show how various forms of damage in a structure will also change the modal properties.

77. Richardson specifically relates some examples of how boundary

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* M. H. Richardson. 1980 (Apr). "Detection of Damage in Structures from Changes in Their Dynamic (Modal) Properties--A Survey," Designation NUREG/CR-1431 UCRL-15103, Lawrence Livermore Laboratory and Structural Measurement Systems, Incorporated; prepared for U. S. Nuclear Regulatory Commission.

condition changes affect the modal properties (Hudson*). A study of a nine-story building showed that the pre-earthquake fundamental frequency was 1.54 Hz. During an earthquake the frequency shifted to 1.0 Hz; following the earthquake the fundamental frequency was 1.27 Hz, but no "apparent" damage occurred to the structure. Tests since the earthquake indicate that the frequency is returning to 1.54 Hz. They also mention that, "The subsequent migration of modal frequencies back to their pre-earthquake values indicates that some material property (perhaps the stiffness of the foundation plus surrounding soil) has returned to its original state."

78. From the report: "From theory to field experience with the non-destructive vibration testing of piles (Davis and Dunn**), we find another example of the evaluation of boundary conditions by the measurement of some dynamic parameters." Although not directly modal information, it is additional information obtained along with the modal measurements. The authors report that the base condition of the pile can be determined by making a low-frequency dynamic stiffness measurement. They have found that an infinitely rigid base will give a maximum stiffness value while an infinitely compressible base will give a minimum stiffness value. All other conditions between a strong anchorage and a weak anchorage will lie between these two values. Davis and Dunn comment: "The dynamic stiffness (E') corresponds to the slope of the initial tangent modulus to a load displacement graph obtained from a static load test on the pile. Although it cannot be pretended that the measurement of E' can be precise, it does give a good indication of the ability of a pile to carry load." They also see, in their measurements, that the denser the soil around the pile, the shorter the average vertical distance between the peaks and valleys of the resonant frequencies (fundamental with various harmonics). This is another case, this

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* D. E. Hudson. 1977 (Dec). "Dynamic Tests of Full-Scale Structures," Journal of the Engineering Mechanics Division, American Society of Civil Engineers, Vol 103, No. EM 6, pp 1141-1157.

** A. G. Davis and C. S. Dunn. 1974. "From Theory to Field Experience with the Non-Destructive Vibration Testing of Piles," Proceedings, Institution of Civil Engineers, Part 2, Vol 57, pp 571-593.

time related to modal measurements (damping), where boundary conditions of a structure can be evaluated by dynamic measurements. The measurement technique consists of a simultaneous force and acceleration measurement.

79. The renewed interest in modal analysis has been sparked by the implementation of efficient computational algorithms, such as the fast Fourier transform (FFT), and in the new mini and micro computers. Better testing and data processing methods, using the new digital equipment, mean a significant savings in time and money.

80. An impact from a hammer introduces the energy into the structure to excite the various modes of interest. The energy from an impact causes many frequencies to be excited simultaneously. The traditional analog technique was time-consuming, because each frequency was excited at different times, as the shaker was swept from one end of the frequency range to the other. Thus the digital equipment, being faster and making modal analysis measurements more easily, holds promise for understanding the complex relationship between boundary conditions and the changes in modal properties.

81. The machine will use the force and acceleration signals to develop the inertance transfer function. It is the ratio of the acceleration to the force in the frequency domain. The mode shape is obtained by assembling the peak values and directions of the imaginary part of the transfer function at the same frequency from all measurements. The modal frequency is simply the location of the imaginary peak along the frequency axis. The width of the modal peak is related to the damping of the mode. That is, the wider the peak, the higher the modal damping. The purpose of modal testing, then, is to artificially excite a structure so that the frequencies, damping, and mode shape of its predominant modes of vibration can be identified.

82. The best measurement that researchers have made in the past, to test the integrity of machinery and various equipment, has been to measure the vibration levels. Because modal analysis is a more complex measurement of the structural response of a structure, it holds promise for developing new techniques to evaluate structures in situ and nondestructively.

Experimental Tests and Results

83. Numerous tests were made on a small concrete beam to investigate the relationship of boundary condition changes with changes in the modal properties. Frequency and damping were measured for the fundamental flexural mode in the free-free condition. The concrete beam was placed on metal, wood, soil, and Plexiglas, and was also suspended in air to test various boundary conditions. The tests made clear that the damping does increase significantly from an unsupported condition, as in air, to a supported condition. The damping of the beam supported at the nodes was 0.473 percent of critical. The specimen was supported by two wooden rods. When the rods were moved to the ends of the specimen, the damping increased to 0.638 percent. The frequency increased from 2451 to 2466 Hz. The damping increased 35 percent and the frequency change was less than 1 percent change.

84. For structures having only partial base support and excited by impact loads parallel to the base, the resonant frequencies did not change significantly. In fact, for a 28,000-lb structure, the resonant frequency was not changed from the position of being supported by the floor to completely suspended in air.

85. For a structure on a foundation, even if there is some bond, the natural frequency does not change for a wide variation in foundation conditions. This means that any change in resonant frequency for lock and dam structures with only base support will reflect changes in the structure.

86. Tests made on the Richard B. Russell Dam in Georgia showed that the damping was about 3 or 4 percent of critical. However, when cylinders (coming from the concrete mixture used for the dam) were tested, it was found that the damping of the specimens was only about 0.37 percent of critical. The specimens were supported at the nodes of the specimen with narrow supports which minimize the loss of energy to the base supports. Although size may have been a factor, the main factor influencing the damping was in the boundary conditions. By embedding a part of the cylinder into soil, the damping increased about 400

percent. With the specimen lying on the soil, the damping was 0.55 percent. When embedded in the soil about 2 in. deep, the damping was 0.86 percent of critical; however, when the specimen was 3 in. deep in the soil, the damping was 1.5 percent. In all cases, the fundamental resonant frequency of the flexural mode was tested. This test indicates the influence of the boundary conditions to influence damping. The frequency was not observed to change significantly.

87. Although no significant change in frequency has been seen for changes made in the base conditions, there have been changes in the frequency when the sides of a structure were restrained. In a test made at Camp Shelby Army Base in Hattiesburg, Mississippi, the walls of a 20-ft-high concrete building were covered with soil. The initial frequency of the wall was 48 Hz with 3.6 percent damping. The frequency increased 38 percent to 66 Hz and the damping increased 133 percent to 8.4 percent of critical when covered with soil. Later, when the soil was removed after the concrete had been subjected to a blast, the frequency went back to 44 Hz and the damping decreased to 2.4 percent of critical. Again, damping was affected more, but here for the first time, we saw a frequency change of considerable proportion.

88. It was found that the damping did change, as the foundation condition changed. The damping was affected by many variables and a consistent relation could not be determined without a more detailed study.

PART V: CONCLUSIONS AND RECOMMENDATIONS

Conclusions

89. Almost all of the old lock and dam structures do not meet current sliding stability requirements and, therefore, require expensive remedial work to strengthen them in sliding stability. Hence, in-place stability evaluations would be very cost-effective. This is especially true, since in all probability many old structures that do not meet the present sliding stability criteria are actually adequate in their resistance to sliding because of inherent safety factors and conservative design assumptions.

90. A good correlation was found between the ratio D_{max}/F_{max} (maximum deflection versus maximum impact force) and the static sliding failure load for various structures and base conditions. This is an ideal relationship, because the ratio of maximum deflection and maximum impact force from a dynamic impact can be obtained for any field structure. The relationship $D_{max}/F_{max} = 2.29 \times 10^{-4}(\text{sliding failure load})^{-1.12}$ gives the static sliding failure load, from which the safety factor follows directly for each case loading.

91. For structures resting on a foundation, their resonant frequency was not significantly affected by a wide variation in boundary conditions. This means that any change in resonant frequency, for lock and dam structures, will reflect changes in the structure for situations where the backfill condition remains constant. In the future, other parameters such as structural damping may be found to have a consistent relation with the condition of a foundation of lock and dam monoliths.

Recommendations

92. The in-place stability relationship should be refined to take into account the variables of impulse of the impact and distance above the base where impacted. The in-place stability relationship should be extended to structures which have more resistance to sliding.

93. The deterioration of a structures foundation condition cannot be studied by using the parameter of resonant frequency, because the base condition does not significantly affect the resonant frequency. Other evaluating parameters, such as structural damping, should be investigated to see if they can be used to reflect the condition of the structure's foundation.

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